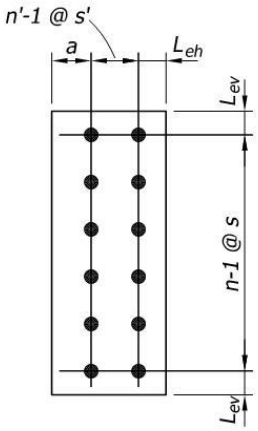
Single angles and plates	$k_a = 1$
Double angles	$k_a = 2$
Connection material length	L_p
Number of Bolts Rows	n
Number of Bolt Columns	n’
Distance from weld line to first bolt column	a
Vertical bolt spacing	s
Horizontal bolt spacing	s’



Bolt Holes in Calculations

- For all hole related limit states except tear out, the effective hole diameter used in calculations is

  $d'_h = d_h + 1/16$ in. $\quad d_b + 1/8$

  The additional 1/16 in. accounts for damage from punching and drilling.

  $d_b + 1/16$

- For tear out, the actual hole diameter is used.

Note: For bearing, the bolt diameter is used. d_b

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Bolts

From Specification Section J.3.2

High-strength bolts in this Specification are grouped according to material strength as follows:

Group 120	ASTM F3125/F3125M Grades A325, A325M, F1852, and ASTM A354 Grade BC
Group 144	ASTM F3148 Grade 144
Group 150	ASTM F3125/F3125M Grades A490, A490M, F2280, and ASTM A354 Grade BD
Group 200	ASTM F3043 and ASTM F3111

Bolt Group	F_{nv} ksi	F_u Tensile Stress ksi
120	54	120
144	108	144
150	113	150
200	150	200

J3.6 *Single bolt shear* $r_v = F_{nv}A_b$ B-1

$$A_b = \frac{\pi d_b^2}{4}$$

$d_b =$ nominal unthreaded bolt diameter

Bolt Shear ϕ = 0.75, Ω = 2.00

Bolt shear w/o eccentricity $R_n = nr_v$ B-2

Manual *Bolt shear w/ eccentricity* $R_n = Cr_v$ B-3

7-30 Bolt shear coefficient **C** from Manual Table 7-6 thru 7-13

Eccentricity $e_b = a$ for tee connections

Refer to Single Plate and Extended Plate Sections for additional eccentricities.

Bearing and Tearout strength at bolt holes

J3.10 w/o eccentricity on bolts

$Bearing = S_2 d_b t F_u$ (J3-6) $Tearout = S_1 l_c t F_u$ (J3-6)

$R_n = k_a(Edge + (n-1)Interior)$ ϕ = 0.75, Ω = 2.00 B-4a

$$Interior = Min\left(S_1\left(s - 1.0\left(d_b + {}^1/_{16}\right)\right), S_2 d_b\right) F_u t$$

$$Edge = Min\left(S_1\left(L_{ev} - 0.5\left(d_b + {}^1/_{16}\right)\right), S_2 d_b\right) F_u t$$

When deformations at bolt holes are design consideration,

$$S_1 = 1.2 \;\&\; S_2 = 2.4$$

When deformations at bolt holes are not a design consideration,

$$\boldsymbol{S_1 = 1.5} \;\&\; \boldsymbol{S_2 = 3.0}$$, used at SPSC

At long slotted holes, $S_1 = 1.0 \;\&\; S_2 = 2.0$

J3.10 w eccentricity on bolts

$R_n = k_a C * min(Edge, Bearing)$ ϕ = 0.75, Ω = 2.00 B-4b

Minimum Bolt Transfer Strength at bolt holes (User Note J3.7)

Top Hole at Uncoped Beams:

k_a*Bolt Shear

k_a*Bearing on Connection Material

Use Minimum value of: k_a*Connection Material Non-edge Tearout

k_a*Bearing on Beam Web

k_a*Beam Web Non-edge Tearout

Middle Holes at Coped and Uncoped Beams:

k_a*Bolt Shear

k_a*Bearing on Connection Material

Use Minimum value of: k_a*Connection Material Non-edge Tearout

k_a*Bearing on Beam Web

k_a*Beam Web Non-edge Tearout

Bottom Holes at Uncoped Beams:

k_a*Bolt Shear

k_a*Bearing on Connection Material

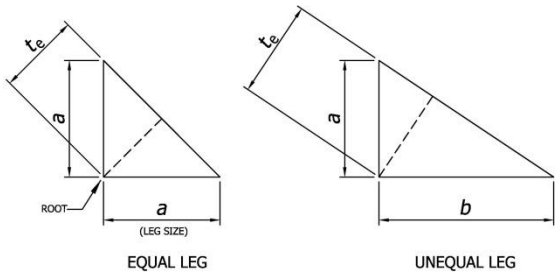
Use Minimum value of: k_a*Connection Material Edge Tearout

k_a*Bearing on Beam Web

k_a*Beam Web Non-edge Tearout

Welds

Fillet Welds (F_{EXX} = 70 ksi)



EFFECTIVE THROAT DIMENSIONS FOR FILLET WELDS

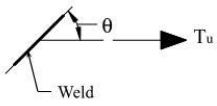
For equal legged Fillet Welds, $A_w = \frac{\sqrt{2}}{2}aL$ Where *L* = Weld Length

For unequal legged Fillet Welds, $A_w = \frac{ab}{\sqrt{a^2 + b^2}}L$

To allow weld sizes in D/16, $A_w = \frac{\sqrt{2}}{2}\frac{D}{16}L$ =

J2. *Welds* J2.4. *Design Strength*

Design Strength = $\phi F_w A_w$.



For Fillet Welds

ϕ = 0.75

$F_w = 0.60 F_{EXX}(1.0 + 0.50 \sin^{1.5}\theta)$

F_{EXX} = electrode strength, ksi

θ = angle of loading measured from the weld longitudinal axis, degrees = (angle of attack)

J2.4. *Fillet Welds Direct shear (no eccentricity)*

$$R_n = 0.6F_{EXX}\frac{\sqrt{2}}{2}\frac{D}{16}L(1.0 + 0.5\,sin^{1.5}\,\theta) = 1.856DL(1.0 + 0.5\,sin^{1.5}\,\theta)$$

ϕ = 0.75, Ω = 2.00

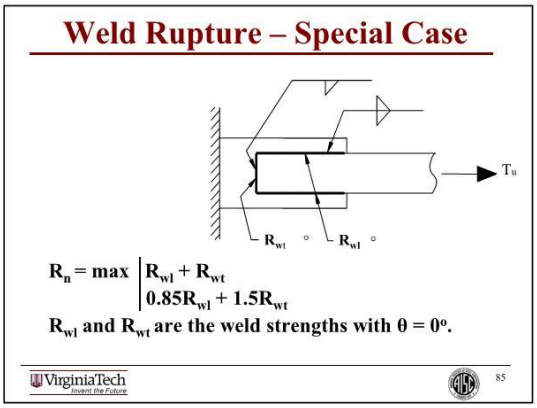
$$\phi R_n = 1.392DL(1.0 + 0.5\,sin^{1.5}\,\theta) \quad \textit{LRFD} \qquad \text{W-1a}$$

$$\frac{R_n}{\Omega} = 0.928DL(1.0 + 0.5\,sin^{1.5}\,\theta) \quad \textit{ASD} \qquad \text{W-1b}$$

Base Metal Effective Weld Sizes:

Shear Rupture $\phi_w 0.60F_{EXX}\left(\frac{D}{16\sqrt{2}}\right) = \phi_{BM}0.60F_u t$

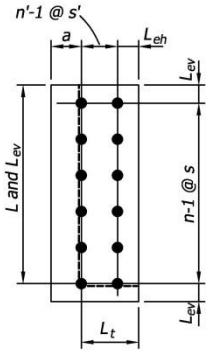
AISC Spec or Manual Section			Connections Formula Number
		$D_{eff} = 22.6\left(\frac{F_u}{F_{EXX}}\right)t = 22.6\frac{F_u t}{70} = \frac{F_u t}{3.09}$ for 70ksi weld	
	For welds on one side	$\left(D_{eff}\right)\frac{F_u t}{3.09_{max}}$	W-2
		$t = \frac{3.09D}{F_u}$ for welds on one side of support	
	When support	$t < \frac{3.09\ D}{F_u}$, reduce weld capacity by $\frac{F_u t}{3.09\ D}$	
	For weld on both sides	$\left(D_{eff}\right)\frac{F_u(2t)}{3.09}\ \frac{F_u t}{6.19_{max}}$	W-3
		$t = \frac{6.19\ D}{F_u}$ for welds on both sides of support	
	When support	$t < \frac{6.19\ D}{F_u}$, reduce weld capacity by $\frac{F_u t}{6.19\ D}$	
	Shear with eccentricity by ultimate analysis		
Manual Page 8-9		$R_n = CDL$	W-4
		C from AISC Table for single angle	
		$e_w = a + e_b$	
		C from AISC Table	
	Shear with eccentricity by vector analysis		
	Double angle clips at	$R_n = \frac{0.6F_{EXX}\frac{\sqrt{2}}{2}\frac{D}{16}(2L)}{\sqrt{1+\frac{12.96e^2}{L^2}}}$	
	Web Bolt-Weld to support	$R_n = 2 * \frac{1.856DL}{\sqrt{1+\frac{12.96e^2}{L^2}}}$ ϕ = 0.75, Ω = 2.00	W-5
		Where e = width of column leg of angle	
	Unstiffened seat angle	$R_n = \frac{0.6F_{EXX}\frac{\sqrt{2}}{2}\frac{D}{16}(2L)}{\sqrt{1+\frac{20.25e_w^2}{L^2}}}$	
		$R_n = 2 * \frac{1.856DL}{\sqrt{1+\frac{20.25e_w^2}{L^2}}}$ ϕ = 0.75, Ω = 2.00	W-6
		Where L = width of column leg of angle	
		e_w = eccentricity from column face to shear resultant	
	Stiffened seat	$R_n = \frac{0.6F_{EXX}\frac{\sqrt{2}}{2}\frac{D}{16}(2.4L^2)}{\sqrt{16e^2+L^2}}$	
		$R_n = \frac{4.454DL}{\sqrt{1+\frac{10.2W^2}{L^2}}}$ ϕ = 0.75, Ω = 2.00	W-7
		Where L = width of column leg of angle	
		e = 0.8W and	
		each horizontal leg length = 0.2L	



Connection Material

J4.1(a)	Tensile yield strength	$R_n = F_y A_g = k_a F_y L_p t$	ϕ = 0.90, Ω = 1.67	CM-1
J4.1(b)	Tensile rupture strength	$R_n = F_u A_e = k_a F_u \left(L_p - n\left(d_b + \frac{1}{8}\right)\right)t$	ϕ = 0.75, Ω = 2.00	CM-2
J4.2(a)	*Shear yield strength*	$R_n = 0.60 F_y A_{gv} = k_a \left(0.6F_y\right) L_p t$	ϕ = 1.00, Ω = 1.50	CM-3
J4.2(b)	*Shear rupture strength*	$R_n = 0.60 F_u A_{nv} = k_a (0.6F_u)\left(L_p - n\left(d_b + \frac{1}{8}\right)\right)t$	ϕ = 0.75, Ω = 2.00	CM-4
J4.3	*Block shear strength*			
	Block shear length	$L = s(n-1) + L_{ev}$		CM-5
	Gross shear area	$A_{gv} = tL$		
	Net shear area	$A_{nv} = t(L - (n - 0.5)d_h)$		
	Net tension area	$A_{nt} = t(L_{eh} - 0.5d_h)$		

Divide by t gives



L (Vertical length of shear plane) CM-5a

$L_s = L - (n - 0.5)\left(d_b + \frac{1}{8}\right)$ CM-5b

$L_t = L_{eh} - 0.5\left(d_b + \frac{1}{8}\right)$ (Less 0.25 at beam ends) CM-5c

$R_n = k_a t(0.6F_U L_s + U_{bs} F_u L_t)$ ϕ = 0.75, Ω = 2.00 CM-5d

Where: $0.6F_u L_s \le 0.6F_y L$ and

$U_{bs} = 1.0$ for all except for multiple column connections

Unstiffed Angle Seat Flexural Yielding

$R_n = F_y \frac{L t_a^2}{4e} \le \left(0.6F_y\right) L t_a$ ϕ = 0.90, Ω = 1.67 CM-6

$e = \frac{l_b}{2} + Setback - \frac{1}{4} - t_a - r$

AISC Spec or Manual Section			Connections Formula Number
		where l_b is largest required bearing length from Equations CM-7, CM-8, CM-9 & CM-10 (cannot exceed width of angle leg minus ¾-in.)	
		r = angle fillet radius taken as $^3/_8$-in.	
		t_a = angle thickness	
J10.2	*Local web yielding*	$R_n = (2.5k + l_b)F_{yw}t_w$	
		l_b = Bearing Length	
		Solving for l_b gives	
		$l_b = \frac{R_n - 2.5kF_{yw}t_w}{F_{yw}t_w}$	
		$R_1 = 2.5kF_{yw}t_w$	
		$R_2 = l_bF_{yw}t_w$ Where l_b=1	
		$l_{b,min}$ $\frac{R - (\phi,\Omega)R_1}{(\phi,\Omega)R_2}$ ϕ = 1.00, Ω = 1.50	CM-7
		$l_{b,min}$	CM-8
	Web crippling when $\frac{l_b}{d} \le 0.2$		
J10.3(b)(1)		$R_n = 0.40t_w^2\left[1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}Q_f$	
		Where Qf = 1 for wide-flange members and HSS wall in tension; Table K3.2 for all other HSS.	
		Solving for l_b gives	
		$l_b = \frac{R_n - 0.40t_w^2\sqrt{\frac{EF_{yw}t_f}{t_w}}}{0.40t_w^2\left(\frac{3}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\sqrt{\frac{EF_{yw}t_f}{t_w}}Q_f}$	
		$R_3 = 0.40t_w^2\sqrt{\frac{EF_{yw}t_f}{t_w}}Q_f$	
		$R_4 = 0.40t_w^2\left(\frac{3}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\sqrt{\frac{EF_{yw}t_f}{t_w}}Q_f$	
		$l_{b,min}$ $\frac{R - (\phi,\Omega)R_3}{(\phi,\Omega)R_4}$ ϕ = 0.75, Ω = 2.00	CM-9
	Web crippling when $\frac{l_b}{d} > 0.2$		
J10.3(b)(2)		$R_n = 0.40t_w^2\left[1 + \left(\frac{4l_b}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}Q_f$	
		Where Qf = 1 for wide-flange members and HSS wall in tension; Table K3.2 for all other HSS.	
		Solving for l_b gives	
		$l_b = \frac{d}{4}\left[\frac{R_n - 0.40t_w^2\sqrt{\frac{EF_{yw}t_f}{t_w}}}{0.40t_w^2\left(\frac{t_w}{t_f}\right)^{1.5}\sqrt{\frac{EF_{yw}t_f}{t_w}}} + 0.2\right]$	

AISC Spec or Manual Section				Connections Formula Number
		$R_5 = 0.40t_w^2\left(1-0.2\left(\frac{t_w}{t_f}\right)^{1.5}\right)\sqrt{\frac{EF_{yw}t_f}{t_w}}$		
		$R_6 = 0.40t_w^2\left(\frac{4}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\sqrt{\frac{EF_{yw}t_f}{t_w}}$		
		$l\frac{R-(\phi,\Omega)R_5}{(\phi,\Omega)R_6}_{b,min}$	ϕ = 0.75, Ω = 2.00	CM-10
F11	*Tee Stem Flexural Yielding*	$R_n = F_y\frac{tL_p{}^2}{4a} \le 1.6F_y\frac{tL_p{}^2}{6a}$	ϕ = 0.90, Ω = 1.67	CM-3
	Tee Stem Flexural rupture	$R_n = \frac{F_uZ_{net}}{a}$	ϕ = 0.75, Ω = 2.00	CM-4

Stiffened Seated Connections:

HSS Wall Yield Line $R_n = k\frac{t^2F_yL}{4e}$ ϕ = 0.90, Ω = 1.67 CM-11

Where B = HSS Connection face width

$e = 0.8W$

$k = f[gh + m + n]$

$f = \frac{2}{2B - 0.4L - t_s}$

$g = \left(1 + \frac{B\sqrt{7}}{4L}\right)$

$h = \sqrt{(B - 0.4L - t_s)(7B + 0.4L + t_s)}$

$m = \frac{B(B - 0.4L - t_s)}{4L}$

$n = 2L + B\sqrt{7}$

(Abolitz and Warner, AISC Engineering Journal, 1st Qtr 1965)

Column Web Yield Line $R_n = k\frac{t^2F_yL}{4e}$ ϕ = 0.90, Ω = 1.67 CM-12

Where T = Clear distance between web fillets

$e = 0.8W$

$k = A[CD + E + G]$

$A = \frac{2}{2T - 0.4L - t_s}$

$C = \left(2 + \frac{T\sqrt{3}}{2L}\right)$

$D = \sqrt{(T - 0.4L - t_s)(3T + 0.4L + t_s)}$

$E = \frac{T(T - 0.4L - t_s)}{2L}$

$G = 4L + 2T\sqrt{3}$

(Ellifritt and Sputo, *AISC Engineering Journal*, 4th Qtr 1999)

AISC Spec or Manual Section				Connections Formula Number

Main Material

Beam Web

J4.1(a)	Tensile yield strength	$R_n = F_y A_g = k_a F_y L_p t$	ϕ = 0.90, Ω = 1.67	MM-1
J4.1(b)	Tensile rupture strength	$R_n = F_u A_e = k_a F_u \left(L_p - n\left(d_b + \frac{1}{8}\right)\right)t$	ϕ = 0.75, Ω = 2.00	MM-2
J4.2(a)	*Shear yield strength*	$R_n = 0.60 F_y A_{nv} = 0.6 F_y L_p t$	ϕ = 1.00, Ω = 1.50	MM-3
J4.2(b)	*Shear rupture strength*	$R_n = 0.60 F_u A_e = 0.6 F_u \left(L - n\left(d_b + \frac{1}{8}\right)\right)t$	ϕ = 0.75, Ω = 2.00	MM-4
	Beam web strength at weld	$R_n = \frac{0.6 F_u t_w (1.0)}{1.856(1.5) 2D(1.0)} WRS$ at double-angles		MM-5

Where *WRS* = Calculated Weld Rupture Strength

Flange Stiffening

J10.1	*Flange local bending*	$R_n = 6.25 F_{yf} {t^2}_f$	ϕ = 0.90, Ω = 1.67	
		$P_{fb} = 6.25 F_{yf} {t^2}_f$		MM-6

Web Stiffening

J10.2(b)	*Local web yielding*	$R_n = (2.5k + l_b) F_{yw} t_w$	ϕ = 1.00, Ω = 1.50	

Where l_b = Bearing Length

$R_1 = 2.5 k F_{yw} t_w$

$R_2 = F_{yw} t_w$

Load applied:

$\leq d$ from end	$(\phi,\Omega) R_n = (\phi,\Omega) R_1 + l_b (\phi,\Omega) R_2$	MM-7
$> d$ from end	$(\phi,\Omega) R_n = 2(\phi,\Omega) R_1 + l_b (\phi,\Omega) R_2$	MM-8

J10.3(b)(1) *Web local crippling when* $\frac{l_b}{d} \leq 0.2$ ϕ = 0.75, Ω = 2.00

$$R_n = 0.40 t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E F_{yw} t_f}{t_w}} Q_f$$

Where Qf = 1 for wide-flange members and HSS wall in tension;
Table K3.2 for all other HSS.

$$R_3 = 0.40 t_w^2 \sqrt{\frac{E F_{yw} t_f}{t_w}}$$

$$R_4 = 0.40 t_w^2 \left(\frac{3}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5} \sqrt{\frac{E F_{yw} t_f}{t_w}}$$

$(\phi,\Omega) R_n = (\phi,\Omega) R_3 + l_b (\phi,\Omega) R_4$ MM-9

J10.3(b)(2) *Web local crippling when* $\frac{l_b}{d} > 0.2$ ϕ = 0.75, Ω = 2.00

$$R_n = 0.40 t_w^2 \left[1 + \left(\frac{4N}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E F_{yw} t_f}{t_w}}$$

AISC Spec or Manual Section			Connections Formula Number
		Solving for l_b gives	
		$R_5 = 0.40t_w^2\left(1 - 0.2\left(\frac{t_w}{t_f}\right)^{1.5}\right)\sqrt{\frac{EF_{yw}t_f}{t_w}}$	
		$R_6 = 0.40t_w^2\left(\frac{4}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\sqrt{\frac{EF_{yw}t_f}{t_w}}$	
		$(\phi,\Omega)R_n = (\phi,\Omega)R_5 + l_b(\phi,\Omega)R_6$	MM-10
J10.3(a)	*Web local crippling when* load applied $> d$ from end	ϕ = 0.75, Ω = 2.00	
		$R_n = 0.40t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}$	
		$(\phi,\Omega)R_n = 2[(\phi,\Omega)R_3 + l_b(\phi,\Omega)R_4]$	MM-11
Web Compression buckling			
J.10.8		$R_n = \frac{24t_w^3\sqrt{EF_{yw}}}{h}$ ϕ = 0.90, Ω = 1.67	
		$P_{wb} = \frac{24t_w^3\sqrt{EF_{yw}}}{h}$ ϕ = 0.90, Ω = 1.67	MM-12
Coped Beam Strength			
	Coped Section	$M = R_n e = F_{cr}S_{net}$	
		$R_n = \frac{F_{cr}S_{net}}{e}$ ϕ = 0.90, Ω = 1.67	MM-13
	Where	$S_{net} = \frac{I_{tr}}{N_{comp}}$ for single coped beams and	
		$\frac{bd^2}{6}$ for double coped beams	
		$e = a - L_{eh} + c + 0.25$	
	Top flange coped only		
		$F_{cr} = \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t_w}{h_o}\right)^2 fk \le F_y$	
		$F_{cr} = 26210\left(\frac{t_w}{h_o}\right)^2 fK$	MM-12a
		$f = \frac{2c}{d}$ when $\frac{c}{d} \le 1.0$	
		$f = 1 + \frac{c}{d}$ when $\frac{c}{d} > 1.0$	
		$k = 2.2\left(\frac{h_o}{c}\right)^{1.65}$ when $\frac{c}{h_o} \le 1.0$	
		$k = 2.2\left(\frac{h_o}{c}\right)$ when $\frac{c}{h_o} > 1.0$	
		c = cope length	
		d_c = cope depth	
		$h_o = d - d_c$	
	Both Flanges coped same lengthand c ≤ 2d, d_c < 0.2d		
		$F_{cr} = 0.62\pi E\left(\frac{t_w^2}{ch_o}\right)f_d \le F_y$	

$$F_{cr} = 56485\left(\frac{t_w^2}{ch_o}\right)f_d$$ MM-12b

$$f_d = 3.5 - 7.5\left(\frac{d_c}{d}\right)$$

All other cope conditions

$$F_{cr} = F_y Q$$ MM-12c

$Q = 1$ for $\lambda \leq 0.7$

$Q = 1.34 - 0.486\lambda$ for $0.7 < \lambda \leq 1.41$

$Q = \frac{1.30}{\lambda^2}$ for $\lambda > 1.41$

$$\lambda = \frac{h_o\sqrt{F_y}}{10t\sqrt{475 + 280\left(\frac{h_o}{c}\right)^2}}$$

At cope conditions where tension flange is longer than compression flange

S_{net} = *net elastic section modulus at end of tension flange*

Ductility

Local buckling $t\frac{L_p}{234}\sqrt{\frac{F_y}{K}}_{min}$ D-1

$$K = \frac{6}{\pi^2}\left[(1-\nu) + \frac{\left(\frac{\pi bm}{a}\right)^2}{6}\right]$$

Where: $b = \frac{L_p}{2}$

$m = 1$

$\nu = 0.3$, Poisson' ratio

Tee Stem Ductility

Min. weld size at support $w\frac{F_{yc}t_f^2}{b}\left(\frac{b^2}{L^2} + 2\right)\frac{5}{8}_{s_{min}}$ D-2

Where t_f = tee flange thickness

b = $(b_f - k_1)/2$ of tee

t_s = tee stem thickness

Max. Tee stem thickness $t_s = \frac{d_b}{2} + \frac{1}{16}$ D-3

Single-Plate

HSS Column Checks

K1.2 $F_y \leq 52\ ksi$

$\frac{F_y}{F_u} \leq 0.8$

AISC Spec or Manual Section				Connections Formula Number
	HSS wall cannot be slender	$\frac{b}{t} \le 1.40\sqrt{\frac{E}{F_y}}$		
10-10	*HSS Punching Shear*	Re $\le \frac{F_u t_{des} l_p^2}{5}$		
		$\boldsymbol{t_{des} \ge \frac{5Re}{F_u l_p^2}}$	ϕ = 0.75, Ω = 2.00	SP-1

Eccentricity & Rotational Ductility

n	Hole Type	e_b	Max t_w or t_p
2-5	SSLT	$\frac{a}{2}$	None
	STD	$\frac{a}{2}$	$\frac{d_b}{2}+\frac{1}{16}$
6-12	SSLT	$\frac{a}{2}$	$\frac{d_b}{2}+\frac{1}{16}$
	STD	a	$\frac{d_b}{2}-\frac{1}{16}$

SP-2

Weld $w = \frac{5}{8}t_p$ SP-3

Extended Single-Plate

Maximum plate thickness $t \frac{6M_{max}}{F_y L_p^2}_{max}$ ESP-1

Where $M \frac{F_v}{0.90} b_{max}$

Exceptions:
a. For single bolt column, ignore if
either t_p or $t_w \le \frac{d_b}{2}+\frac{1}{16}$ and both $L_{eh} \ge 2d_b$
b. For double bolt column, ignore if
both t_p or $t_w \le \frac{d_b}{2}+\frac{1}{16}$ and both $L_{eh} \ge 2d_b$

Shear & flexure interaction (Flexural Yielding) $R_n = \frac{F_y L_p t_p}{\sqrt{2.25 + 16\left(\frac{a}{Lp}\right)^2}}$ ϕ = 0.90, Ω = 1.67 ESP-2

Plate buckling $R_n = \frac{F_{cr} S_{net}}{a}$ ϕ = 0.90, Ω = 1.67 ESP-3

AISC Spec or Manual Section		Connections Formula Number
	$F_{cr} = F_y Q$	
	$Q = 1$ for $\lambda \le 0.7$	
	$Q = 1.34 - 0.486\lambda$ for $0.7 < \lambda \le 1.41$	
	$Q = \frac{1.30}{\lambda^2}$ for $\lambda > 1.41$	
	$\lambda = \frac{L_p\sqrt{F_y}}{10t_p\sqrt{475 + 280\left(\frac{L_p}{a}\right)^2}}$	
Weld	$w = \frac{5}{8}t_p$	ESP-4
Stabilizer Plate Requirements	If applied load $\le \frac{1500L_p t_p^3}{a^2}$ ϕ = 0.90, Ω = 1.67	ESP-5
	No stabilizer plates required	
Connection Torsion Strength	Not applicable if metal deck supports top flange	
	$M_T = R\left(\frac{t_w + t_p}{2}\right)$	
	$M_T \le \left[(\phi_v,\Omega_v)0.6F_{yp} - \frac{R}{L_p t_p}\right]\frac{L_p t_p}{2} + \frac{2R(t_w + t_p)b_f}{(\phi_b,\Omega_b)F_{yb}L_s t_w^2}$	ESP-6
	Where L_s= Beam span in inches	
Limit States of Shear Yielding & Flexural Yielding		
	$\left(\frac{V_r}{(\phi,\Omega)V_n}\right)^2 + \left(\frac{M_r}{(\phi,\Omega)M_n}\right)^2 \le 1$	ESP-7
Flexural LTB		
F.11.2.(a)	If $\frac{L_b d}{t^2} \le \frac{0.08E}{F_y}$, then LTB does not apply	
F11.2.(b)	If $\frac{0.08E}{F_y} < \frac{L_b d}{t^2} < \frac{1.9E}{F_y}$,	
	$M_n = C_b\left[1.52 - 0.274\left(\frac{L_b d}{t^2}\right)\frac{F_y}{E}\right]M_y \le M_p$	
	Where $C_b = 1.84$	
F11.2.(c)	If $\frac{L_b d}{t^2} > \frac{1.9E}{F_y}$,	
	$M_n = F_{cr}S_x \le M_p$	
	$R_n = \frac{M_n}{a}$	ESP-8